DESIGN OF A 207 FT. SPAN SPANDREL-BRACED TWO-HINGED ARCH

BY

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Armour Institute of Technology

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Design of a 207 ft. span
spandrel braced two-hinged

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DESIGN OF

207' SPAN SPANDREL-BRACED

TWO-HINGED ARCH

A THESIS

Presented by

Roe L. Stevens

Wm Trinkaus, Jr.

To the

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Prof Civil Engineering

H.M. Raymond

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The Two-Hinged Spandrel-Braced Steel Arch.

The arch is a structure so arranged that the supporting forces which result from loads are not parallel to the direction of action of the loads, but usually act in the nature of thrusts on the springings of the arch. In arches of masonry, concrete, or steel in the form of ribs, the arch element is usually a linear curved member or members so formed that the center of pressure on any section, due to loads, lies within the rib. The arch ring or rib itself is all that is subjected to arch action; the material above, consisting of spandrels and filling, is so formed that it does not assist in resisting the bending stresses on the arch but acts only in imposing loads on the arch. This condition is more perfectly realized where the platform or filling is supported by posts or arches resting on the arch ring.

In the spandrel-braced arch not only does the lower curved member take the arch action but the entire truss acts as an arc in much the same manner as the ordinary truss acts as a beam.

Previous [illegible]
in any other kind of structure, the supporting forces of reactions caused by the loads on the structure, must be found. In a beam or truss not restrained from moving laterally at the supports the reactions are parallel to the loading forces, usually vertical, and are found from the simple conditions of static equilibrium.

With the arch, there is, in addition to these vertical reactions, a thrust upon the abutments so that the reactions are inclined and are the resultants of the same vertical reaction which exist on a beam similarly loaded, combined with those directly opposing the horizontal thrust.

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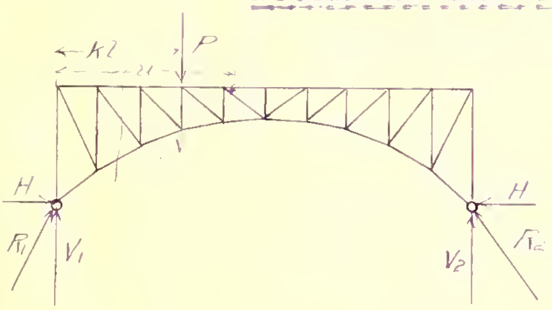
In arches of three hinges, the horizontal thrusts are easily found. The hinges provide three points at which the resistance of the arch section to bending is zero. Since there are three components of the reactions to be found - the vertical reaction at each end and the horizontal thrust - we can write equations for the bending moment on the arch in terms of the loads and reactions, at each hinge; set them each equal to zero, since at these points the arch cannot take bending stresses; and from them solve for the three components of the reactions.

In arches of two hinges there are but two such points at which the bending moments must be equal to zero, so that the conditions of static equilibrium furnish but two equations for solving the three unknown quantities. The other equation then has to be supplied by the elastic deformation of the arch and may be derived in a number of ways.

The formulas for the horizontal thrust on an arch are usually expressed in terms of the moment of interia of the combined section and other quantities. In an arch rib consisting of a linear ring subjected to arch action, the structure above acting simply as a load on the arch ring, this may be assumed as constant or as varying as a function of the inclination of the arch, and, as it occurs in summation in both numerator and denominator of the expression for the horizontal thrust, its actual value does not require to be known. The condition makes it possible to solve directly for the stresses in the sections and so to design them. In a spandrel-braced arch the moment of inertia does not vary in such a way that it can be formulated. When it is not possible so to eliminate the moment of inertia of the section, no direct

solution for the stresses in the members can be made without first knowing the moment of inertia of each section, which in its turn requires that the sections shall have been previously determined. From this it is apparent that the design of an arch of variable section, of which the spandrel-braced arch is an example, par excellence,-cannot be made directly but can only be done by methods of successive approximation, i.e, determining the sections by some roughly approximate method, then using the section areas so obtained in the true formulas from which another set of stresses and areas will be obtained which approach more nearly to the true ones. This process may be repeated until any desired degree of accuracy may be secured.

Determination of the Reactions.



Let the arch be as represented in the figure and be loaded with a single load, P acting at a distance kl from the left hand abutment.

Since the hinges are fixed in position the load will produce two reactions, R_1 and R_2 inclined somewhat as shown. These may be resolved into vertical and horizontal components, of which V_1 and V_2 and H are the vertical and horizontal components respectively. The amount of H at each end is evidently the same, for if they were not equal the horizontal forces acting on the arch would be unbalanced among themselves, which is contrary to the condition of equilibrium assumed.

By taking moments of the external forces about either hinge, it is seen that

$$V_1 = P(1-k) \quad \text{and} \quad V_2 = Pk$$

which is the same as the reactions on a beam of equal span under the same load.

The value of H is the only part of the reactions now undetermined and in the following its value is deduced.

Formula for Horizontal thrust.

The following notation will be used:

P = single vertical load on arch

kl = distance of load from left abutment

R_1 = reaction at left hinge

R_2 = " " right "

V_1 = vertical component of left reaction

V_2 = " " " right "

H = horizontal component of reactions

S_n' = stress which would exist in any member from the vertical components only of the reaction

T_n = stress which would exist in any member from a horizontal reaction of unity

S_n = actual stress in any member from load P

A_n = sectional area of any member

L_n = length of any member

E = modulus of elasticity of steel

δ_n = deformation of any member due to load P

Δ = horizontal deflection of abutment which would take place under P if one end were free to move laterally.

There have come to our notice three general methods for deriving an expression for the horizontal thrust. All depend on the elastic deformation of the arch and simply involve the employment of different methods to arrive at the same result.

The first method employs the principle of Least Work and is taken from the Proceedings of the American Society of Civil Engineers, vol. 10, p.167, where it is ascribed to Mueller-Breslau's, "Graphic-Statics."

This principle is the expression of a law of nature that where the members among which a given applied load is to be distributed are so arranged that there are any number of divisions of the load which will satisfy the conditions of static equilibrium, then the stresses or supporting forces will distribute themselves so that the total work of elastic deformation will be a minimum. To use this method a number of expressions for the work of deformation are written, involving the unknown forces and various known quantities. As many such equations can be written as there are unknown forces to be determined.

The value of the forces to make the work a minimum are obtained by setting the first derivatives of the work with respect to the unknown forces equal to zero and solving for the unknown forces. From this principle two theorems have been deduced by Castigliano: (Hiroi, "Statically Indeterminate Structures")

1 The displacement of the point of application of an external force acting on a body - caused by the elastic deformation of the latter - is equal to the first derivative of the work of resistance preformed in the body, with respect to the force.

II "The partial derivatives of the work of resistance with respect to the statically indeterminate forces which are so chosen that the forces themselves preform no work are equal to zero."

This principle is applied to the derivation of the formula for horizontal thrust as follows:

The work done in any member due to its elastic deformation is,

$$dW = \tfrac{1}{2} S \delta$$

From Hooke's Law, the elastic deformation in a member is,

$$\delta = \frac{SL}{AE}$$

$$\therefore dW = \frac{S^2 L}{2AE}$$

The work of elastic deformation in all the members is,

$$W = \Sigma \frac{S^2 L}{2AE}$$

But

$$S = S' + HT$$

$$\therefore W = \Sigma \frac{(S' + HT)^2 L}{2AE}$$

From Castigliano, Theorem 1,

$$-\Delta l = \frac{dW}{dH}$$

where Δl is the increment of change in span (if unrestrained) due to $\frac{dW}{dH}$, the increment of work of resistance in the truss.

But

$$W = \Sigma \frac{(S'^2 + 2S'TH + T^2H^2)l}{2AE}$$

$$\therefore \frac{dW}{dH} = \Sigma \frac{(2S'T + 2HT^2)L}{2AE}$$

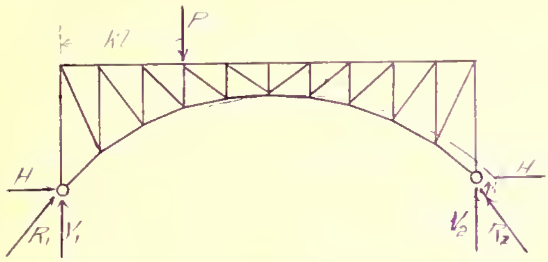
$$\therefore -\Delta l = \Sigma \frac{S'TL}{AE} + H \Sigma \frac{T^2 L}{AE}$$

$$H = -\frac{\Sigma \frac{S'TL}{AE} + \Delta l}{\Sigma \frac{T^2 L}{AE}}$$

If the abutments are fixed in position, $\Delta l = 0$

$$\therefore H = -\frac{\Sigma \frac{S'TL}{AE}}{\Sigma \frac{T^2 L}{AE}}$$

Another method is that given by Merriman and Jacoby, "Roofs and Bridges, Higher Structures."



If we imagine the arch fixed at the left hinge but supported at the right hand end so that it is free to move horizontally when under no load whatever, it may be assumed to be in a position shown by the full lines. If a load P is applied, there will take place certain changes in the lengths of the various members, tending to increase the span and the lower chord will take a position as shown by the dotted line.

If at the right hand abutment we apply a force which shall be just sufficient to force that end back to its original position, we will have duplicated the conditions which exist in the arch under the single load when both hinges are fixed in position and the horizontal force which it was necessary to apply at the right hand hinge is the same as the horizontal thrust in the arch.

If we derive an expression for the deflection of the hinge in the first case, then one for the value of the deflection of the hinge from the application of H, we will have two values for the deflection which are evidently equal, their value can be equated, and from them a value of H. obtained.

To find the deflection of the hinge B under load P imagine a force of any amount, Q to be applied at the point whose deflection is required and in the direction of that deflection. Let Δ' be the deflection at that point from the application of Q, T' the

stress in any member caused by Q and δ the corresponding deformation in the member.

The external work done by Q in causing the deformation Δ' is

$$W = \frac{1}{2} Q \Delta'$$

and the internal work in any member is

$$dW = \frac{1}{2} T' \delta$$

The total internal work of deforming the truss is the sum of the work done on each member, or

$$W = \frac{1}{2} \Sigma T' \delta$$

Since the total internal work = the external work

$$Q \Delta' = \Sigma T' \delta$$

P is the load on the truss which produces the thrust H, the actual stresses S', and the deformation λ, in the members. Δ = actual movement of hinge due to P. External work = $\frac{1}{2} H \Delta$ = internal work = $\frac{1}{2} \Sigma S' \lambda$

And

$$\lambda = \frac{S' L}{A E}$$

The deformation of the truss may be considered as made up of the sum of the parts contributed by the deformation in each member.

Let Δ_1' be the portion of the deflection under the force Q due to the deformation in any member.

$$\therefore \frac{1}{2} Q \Delta_1' = \frac{1}{2} T \delta$$

$$\therefore \Delta_1' = \frac{S}{H} \lambda = \frac{T}{Q} \delta$$

Likewise, let Δ_1 be the portion of the deflection of the abutment under the load P due to the deformation in any member.

$$\therefore \frac{1}{2} H \Delta_1 = \frac{1}{2} S' \lambda$$

$$\therefore \Delta_1 = \frac{S'}{H} \lambda$$

Now for any member, $\frac{S'}{H} = \frac{T}{Q}$ since each term is the stress in the member divided by the force, acting at the same point, which causes the stress. (Stresses in the members are directly proportional to the intensity of the forces causing them.)

Substituting for $\frac{S'}{H}$ its equal, $\frac{T}{Q}$,

$$\Delta_1 = \frac{T}{Q}\lambda = \frac{T}{Q}\frac{S'L}{AE}$$

This is the portion of the deflection of the hinge due to the change in length of one member. The total deflection equals the sum of these portions for the different members, or,

$$\Delta = \Sigma\Delta_1 = \Sigma\frac{T}{Q}\frac{S'L}{AE}$$

Since the actual value of Q is immaterial, let its value be taken as unity, so that

$$\Delta = \Sigma\frac{S'TL}{AE}$$

This gives the deflection of the right abutment outward due to the load P.

We have now to derive another value of Δ from the effect of H in pushing the right hand hinge back to its original position. Let H be the force applied horizontally at the abutment. Δ is the deflection of the hinge. The stress in any member is HT and its deformation, δ.

$\therefore$ External work $= \frac{1}{2}H\Delta$ = internal work $= \frac{1}{2}HT\delta$

$$\therefore \Delta = \Sigma T\delta$$

But

$$\delta = \frac{(HT)L}{AE}$$

$$\therefore \Delta = \Sigma\frac{T^2HL}{AE}$$

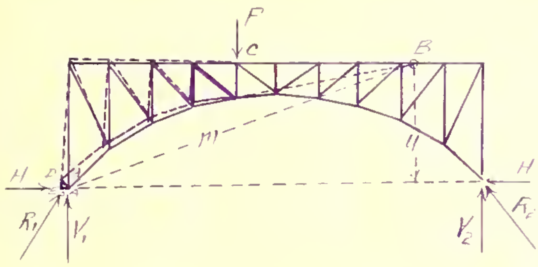
$$= H\Sigma\frac{T^2L}{AE}$$

Equating the two values of Δ obtained,

$$\Sigma \frac{S'TL}{AE} = H \Sigma \frac{T^2 L}{AE}$$

$$\therefore H = \frac{\Sigma \frac{STL}{AE}}{\Sigma \frac{T^2 L}{AE}}$$

Another method for deriving the value of H is that given by Prof. Green in his book on "Arches." When the arch is under load certain deformations will take place in the various members. As a result, if one end were fixed and the other free to move, a certain amount of movement would take place at the free end.



This may be considered [illegible] mation in each member causing a certain portion of the total change in span.

Consider the effect of the change in length of a single member, the other members being unaffected. The free portion of the truss will revolve about a center which is at the intersection of the two members cut by a section passing through the member considered. The amount of motion of any point in the truss is proportional to its distance from this center of rotation.

Let m be the distance from the center of rotation to the free end, and v the distance to the member. Let λ be the deformation in the member and d the distance passed over by the free end, AD.

$$\therefore \frac{d}{m} = \frac{\lambda}{v}$$

Let Δ_1 be the horizontal component of d, or AE, and y the vertical distance from the free end to the center of rotation. Since the direction of motion of the free end is perpendicular to

the radius of its motion, the horizontal component of d makes the same angle with d that y, the vertical component of m, makes with m.

$$\therefore \frac{\Delta_1}{y} = \frac{d}{m} = \frac{\lambda}{v}$$

$$\therefore \Delta_1 = \frac{\lambda}{v} y$$

Let HT be the stress in the member due to H, S' that due to the vertical component of the reaction only, and S the actual stress in the member.

$$\therefore S = S' + HT$$

$$\lambda = \frac{SL}{AE}$$

$$= \frac{S'L}{AE} + \frac{HTL}{AE}$$

$$\therefore \Delta_1 = \frac{\lambda}{v} y = \frac{SL}{AE} \frac{y}{v}$$

$$= \frac{S'L}{AE} \frac{y}{v} + H \frac{TL}{AE} \frac{y}{v}$$

But

$$\frac{y}{v} = T$$

$$\therefore \quad \Delta_1 = \frac{S'TL}{AE} + H \frac{T^2L}{AE}$$

The total change of span,

$$\Delta = \Sigma \Delta_1 = \Sigma \frac{S'TL}{AE} + H \Sigma \frac{T^2L}{AE}$$

With immovable abutments, change of span=0

$$\therefore \Delta = 0 = \Sigma \frac{S'TL}{AE} + H \Sigma \frac{T^2L}{AE}$$

$$\therefore H = \frac{\Sigma \frac{S'TL}{AE}}{\Sigma \frac{T^2L}{AE}}$$

Formulas for Computation.

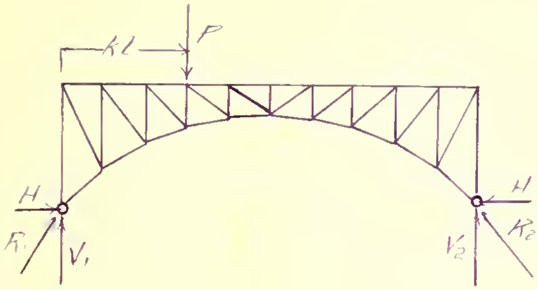
The formulas for the stresses in the members are arranged for computation as follows:

We have the general formula for the horizontal thrust,

$$H = \frac{\Sigma \frac{S'TL}{AE}}{\Sigma \frac{T^2L}{AE}}$$

or, since E is the same for all members,

$$H = \frac{\Sigma \frac{S'TL}{A}}{\Sigma \frac{T^2L}{A}}$$



Let u be the lever arm of the reaction at the nearest abutment, about the center of moments of any member. Let v be the lever arm of the member about its center of moments.

Then for any member to the left of P, the stress in the member, considering the bridge as simply supported,

$$S = V_1 \frac{u}{v}$$

For the corresponding symmetrical member on the right half of the truss,

$$S' = V_2 \frac{u}{v}$$

For the stresses in both members

$$S' = (V_1 + V_2) \frac{u}{v}$$
$$= P \frac{u}{v}$$

For members between the load and the middle of the truss,

$$S' = \frac{1}{v}(V_1 u - P(u - kl))$$
$$V_1 = P(1-k)$$
$$\therefore k = 1 - \frac{V_1}{P}$$
$$\therefore S' = \frac{1}{v}(V_1 u - Pu + Pl - V_1 l)$$

For corresponding members on the right half of the truss,

$$S' = \frac{V_2 u}{v}$$

For the stresses in both members,

$$S' = (V_1+V_2)\frac{u}{v} + \frac{1}{v}(Pl - Pu - V_1 l)$$
$$= P\frac{l}{v} - \frac{Pl}{v} + \frac{Pkl}{v}$$
$$= Pk\frac{l}{v}$$

Since the truss is symmetrical, T, the stress due to horizontal thrust is the same in the symmetrical members.

Substituting for S' in the formula for H

$$H = \frac{\Sigma_0^{kl} P\frac{u}{v}\frac{TL}{A} + \Sigma_{kl}^{\frac{1}{2}l} P\frac{kl}{v}\frac{TL}{A}}{2\Sigma_0^{\frac{1}{2}l}\frac{T^2L}{A}}$$

the summation covering the members in one half the span.

$$\therefore H = P\frac{\Sigma_0^{kl}\frac{u}{v}\frac{TL}{A} + \Sigma_{kl}^{\frac{1}{2}l}\frac{kl}{v}\frac{TL}{A}}{2\Sigma_0^{\frac{1}{2}l}\frac{T^2L}{A}}$$

Let n be the number of the panel point from the left end at which the load P is acting. Let p be the panel length.

Then the distance of the load from the left end,

$$kl = np.$$

$$\therefore H = P\frac{\Sigma_0^{kl}\frac{u}{v}\frac{TL}{A} + n\Sigma\frac{p}{v}\frac{TL}{A}}{2\Sigma\frac{T^2L}{A}}$$

The stress in any member,

$$S = S' + HT$$

Let N be the number of panels in the truss.

$$\therefore l = Np$$
$$V_1 = P\frac{(l-kl)}{l} = P\frac{Np-np}{Np}$$
$$= P\frac{N-n}{N}$$
$$\therefore S' = \frac{V_1 u}{v} = P\frac{N-n}{n}\frac{u}{v}$$

for members to the left of P

and

$$S' = \frac{V_1 u}{V} - \frac{P(u-np)}{V}$$

$$= P\frac{N-n}{N}\,\frac{u}{V} - P\,\frac{u-np}{V}$$

$$P\left(\frac{N-n}{N}\cdot\frac{u}{V} - \frac{u-np}{V}\right)$$

for members between P and the middle of the truss.

Substituting these values of S' and H in the formula for S

$$S = P\left[\frac{N-n}{N}\cdot\frac{u}{V} - \frac{u-np}{V} + \left(\frac{\Sigma_0^{kl}\frac{u}{V}\frac{TL}{A} + n\,\Sigma_{kl}^{\frac{1}{2}l}\frac{p}{V}\frac{TL}{A}}{2\Sigma_0^{\frac{1}{2}l}\frac{T^2L}{A}}\right)T\right]$$

which gives the stress in any member due to load P at panel point n , the second term being dropped, however, except for members between P and the middle of the truss.

The Design.

The bridge selected to be designed according to this method is of the same general dimensions as one designed and built in 1902 by the Chicago, Milwaukee and St. Paul Railroad at Iron Mountain, Michigan, as a three-hinge steel arch. The span is 207' and the depth 52'. It is a single track deck structure with trusses spaced 22 feet center to center.

The bridge crosses the Menominee River and at that point the banks consist of solid granite so that the situation is ideal for an arch span.

For this design the crown depth was assumed as 8 feet, the curve of the lower chord as a parabola, and the span was divided into ten panels of 20.7 feet each. The same loadings, unit stresses and specifications were used so that the two designs serve in a measure as a basis for comparison of the two classes of arches. The outline of one-half of the truss with the lengths and lever arms of the members is given in Plate 1.

The live load is that known as Cooper's "Class E-50 Loading" except that the uniform load following the two locomotives was assumed as 7000 pounds per foot of track instead of 5000 pounds to allow for the excessive weight of ore trains. The intensity of the uniform load was so great that it was used instead of locomotive concentrations in finding the stresses in the trusses. For the floor system the moments and shears were greater for the concentrated loads than for the uniform load. The length of the locomotive (without tender) wheel base being nearly equal to the panel length, the difference between its weight and that of an

equal length of uniform load was taken as an "excess" load and was applied to two such alternate panel points as would produce the maximum stress in each member. The details of the loadings are given in table 1.

The fact that the bridge is anchored only at the abutment hinges, while the largest portion of the trusses and connections and the live loads exposed to wind action are at a considerable distance above the anchorage, gives rise to large overturning momemts which act to produce loads on the truss, acting downward on the leeward side and upward on the windward side.

The distribution of the wind stresses among the various systems of bracing is indeterminate in this kind of a structure, but by making the upper lateral bracing of nominal dimensions we have considered that the loads applied at the upper panel points are carried down through the sway bracing to the lower panel points and from them through the lower system of lateral bracing to the abutments. This represents about the most direct way of transfering the wind loads to the abutments and was taken as the most probable.

The wind on the train was assumed to act 8 feet above the middle of the upper chord and is treated as live load. This live wind load and that applied at the upper chord produces an overturning moment about the corresponding lower chord panel points. Since the lower chord panel points are not in the same horizontal plane, the load at each panel point produces an overturning moment about the next panel point toward the abutment. The loads are given in Table 1. In addition to the vertical

loads on the truss from overturning, the horizontal wind loads acting on the lower chord and transfered to it by the sway frames produce stresses in the lower lateral system. A graphical determination of the stresses in the lower lateral system is given on Plate II and in Table II is given the composition of the lower lateral system.

The design of the intermediate sway bracing is given in Table 111. The stresses in the end sway bracing and in the lattice floor beam are given, graphically, in Plate 1V and the design in Table 1V. The design of the floor system is given in Plate V.

As a preliminary to the computation of the stresses a table of constants for the members of the truss was computed. These are independent of the loads on the truss and are given in Table V.

The stresses in each member due to loads of unity at the various panel points, considering only the effect of the vertical reactions, were next computed. This corresponds to the terms

$$\frac{N-n}{N}\frac{u}{V} - \frac{u-np}{V}$$

The former is given in Table V1 and the latter in Table V11 for the members and loads to which it applies. They are combined in Table V111.

The term

$$\Sigma \frac{u}{V} TL + n \Sigma \frac{p}{V} TL$$

was next computed. Since in this first trial we have nothing to determine the areas of the sections, they are assumed for this purpose to be all equal so that the term cancels out of the expression for H. The values for this expression are given in Table 1X, the quantities for each class of members above the

heavy lines being computed from the first term and those below from the second. Since for panel loads beyond 4 there are no members to the right of the load, then for all succeeding panel loads only the first term applies. The summations are obtained by adding all the quantities for one panel load, all the signs being minus as S' is of opposite sign to T. The summations for each panel load are divided by the quantity,

$$2\sum_0^{12} T^2 L$$

from Table V, and the result is the value of

$$\frac{\sum \frac{u}{v} TL + \sum n \frac{p}{v} TL}{2\sum T^2 L}$$

for each unit panel load in the expression for H.

In Table X these values have been multiplied by the value of T for each member.

In Table X1 these stresses from horizontal reactions are combined with those from vertical reactions in Table V111 and the results are the actual stresses in the members from unit panel loads.

Since for dead load, all panel loads have to be considered as acting at all times and as the same load is concentrated at the corresponding panel points from the two ends, some labor is saved by combining the corresponding unit stresses before multiplying them by the panel loads. These are given in Table X11. Since the live and "excess" panel loads are all equal, the unit stresses which will produce to largest values, plus and minus, are combined in Table X11. In Table X111 are given the stresses due to dead panel loads, being the values in Table X11 multiplied by the panel loads. These are combined and are given in the

column for Dead Load in Table XV. In Table XIV are given the stresses due to wind loads, and under wind loads in Table XV the sums of the stresses of the same sign are given. For the live loads the maximum possible stress of either kind in any member will take place when every panel point which gives stress of that kind is loaded and when all panel points which give stress of the opposite kind are unloaded. In the columns of live load and excess load the quantities in the corresponding columns for unit stresses in Table XII are multiplied by the panel loads. The specified unit stresses provide that the sectional area for live load stresses shall be twice that provided for equal dead load stresses. For this reason the combined maximum and minimum stresses in the last two columns are composed of all the live stresses and half the dead load stresses, and the live load unit stresses used in designing. In Table XVI is given the preliminary design of the members from which areas are obtained to be used in the second trial for stresses. At this point it was thought that to provide for factors which had not been considered in this preliminary, the stresses should be increased about 10%. The results of the second trial indicate that this was not necessary. Since the wind load stresses form so large a proportion of the total stresses, advantage was taken of a provision in the specifications allowing an increase of 30% in the unit stresses used.

From the areas figured, an analysis of dead load was made and was used instead of the original dead load in the subsequent computation. The weights were figured from the areas of the main sections and

to provide for the additional weight of connections an increase of 20 was made. The dead load, in panel weights is given in Table XVII.

In Table XVIII are given the computations of several constants in the formulas, introducing the values of the areas.

The temperature stresses were figured from a modification of the formula for horizontal thrust. In the derivation of the formula for horizontal thrust by the method of Least Work it was found that

$$H = \frac{\Sigma \frac{S'TL}{AE} + \Delta l}{\Sigma \frac{T^2 L}{AE}}$$

where Δl is the change in span.
If for Δl is put the value of the change of span which would take place from the change in length of members from change in temperature if the ends were unrestrained, the resulting value of H would be a thrust upon the abutments caused by that tendency to change of span.

$$\Delta l = \epsilon t l$$

where ϵ is the coefficient of linear expansion per degree Fahrenheit for steel, t the range in temperature above or below the normal and l the span. The term

$$\Sigma \frac{S'TL}{AE}$$

is zero since for horizontal reaction only S' is zero.

The stress in any member due to temperature change will then be the horizontal thrust due to that change multiplied by

The following values for the quantities were taken.

$$E = 29000 \text{ kips}$$

$$\epsilon = .0000065$$

$$l = 207 \text{ feet}$$

$$t = +90\ ,\ -[illegible]0$$

$$\therefore S_t = \frac{29000 \times .0000065 \times 90 \times 207}{2} \cdot \frac{T}{\Sigma \frac{T^2 L}{A}}$$

$$= 175[illegible] \frac{T}{\Sigma \frac{T^2 L}{A}}$$

From Table XV111

$$\Sigma \frac{T^2 L}{A} = 24.665$$

so that $S_t = 7.144\, T$

The values of S_t for each member is given in the last columns in Table XV111.

Tables X1X to XXV1 are similar to those previously mentioned and are self explanatory.

In Table XXV11 is given the detailed designs of the sections as finally adopted and on Plate V1 a general drawing of the arch.

PLATE I. OUTLINE OF ONE-HALF TRUSS

DIMENSIONS OF TRUSS

Member		LENGTH		Lever Arm
Upper Chord	U_1	20.700	20' 8 13/32"	31.160
	U_2	20.700	20' 8 13/32"	23.840
	U_3	20.700	20' 8 13/32"	15.040
	U_4	20.700	20' 8 13/32"	9.760
	U_5	20.700	20' 8 13/32"	8.000
Lower Chord	L_1	26.066	26' 0 7/16"	41.295
	L_2	24.088	24' 1 1/16"	31.074
	L_3	22.493	22' 5 59/64"	21.940
	L_4	21.314	21' 3 49/64"	14.573
	L_5	20.747	20' 8 31/32"	9.738
Verticals	V_0	52.000	52' 0"	67.953
	V_1	36.160	36' 1 59/64"	60.756
	V_2	23.840	23' 10 5/64"	56.079
	V_3	15.040	15' 0 31/64"	58.964
	V_4	9.760	9' 9 1/8"	114.792
	V_5	8.000	8' 0"	
Diagonals	D_1	41.666	41' 8"	58.974
	D_2	31.573	31' 6 7/8"	45.877
	D_3	25.587	25' 7 3/64"	32.961
	D_4	22.885	22' 10 5/8"	24.573
	D_5	22.141	22' 1 11/16"	41.381

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Plate II Loading

Live Loads on One Rail

Weight of 2 locomotives (one rail) 355 000#

Weight of equal length of uniform live load 381 500#

Live panel load 3500 x 20.7 = 79 500#

Maximum live panel load. 112500#

Excess live panel load 33 000#

Total dead load taken as 240 tons = 240000# on each truss

Weight of approach spans (4 panels) 75 tons

Assumed distribution of dead load in per cent of equal panel load

Panel point	0	1	2	3	4	5
% of panel load	177	101	89	83	78	72

Wind Loads

Chicago, Milwaukee & St. Paul R.R. Specification

Upper lateral system — Live Load 450# per lin. ft. acting 8' above upper chord

Dead Load 200# per lin. ft.

Live panel load 450 x 20.7 = 9300#

Dead panel load 200 x 20.7 = 4200#

Lower Lateral system — Dead Load 200# per lin. ft.

Dead panel load 200 x 20.7 = 4200#

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Vertical Loads Due to Overturning from Wind

Table I

Dead Wind Loads

	Upper Chord Panel Loads				Lower Chord Panel Loads			
Panel Point	Load at U_n	Lever Arm about L_n	Moment about L_n	Vertical Load	Load at $L_{(n+1)}$	Lev. Arm about L_n	Moment about L_n	Vertical Load
5	4200	8.00	33600	1500				
4	4200	9.76	41000	1860	8400	1.76	14800	340
3	4200	15.04	63200	2880	12600	5.28	66500	3020
2	4200	23.84	100300	4550	21000	8.80	184700	8400
1	4200	36.16	151700	6910	29400	12.32	362000	16420
0	4200	52.00	218500	9940	37800	15.84	600000	27200

Live Wind Loads

	Upper Chord Panel Loads				Lower Chord Panel Loads			
Panel Point	Load at U_n	Lev. Arm about L_n	Moment about L_n	Vertical Load	Load at $L_{(n+1)}$	Lev. Arm about L_n	Moment about L_n	Vertical Load
5	9300	14.00	130300	5910				
4	9300	17.76	165000	7500	4650	1.76	8200	372
3	9300	23.04	214000	9750	13950	5.28	73700	3350
2	9300	31.84	296000	13460	23250	8.80	204000	9310
1	9300	44.16	410000	18650	32550	12.32	401000	18200
0	9300	60.00	558000	25350	41850	15.84	663000	30150

Summary of Wind Loads

Panel Point	5	4	3	2	1	0
D.L.-U.C.	1500	1860	2880	4550	6910	9940
D.L.-L.C.		340	3020	8400	16420	27200
D.L.-Total	1500	2200	5900	12950	23330	37140
L.L.-U.C.	5910	7500	9750	13460	18650	23550
L.L.-L.C.		372	3350	9310	18200	30150
L.L.-Total	5910	7872	13100	22770	36850	53700
Total	7410	10072	19000	35720	60180	90840

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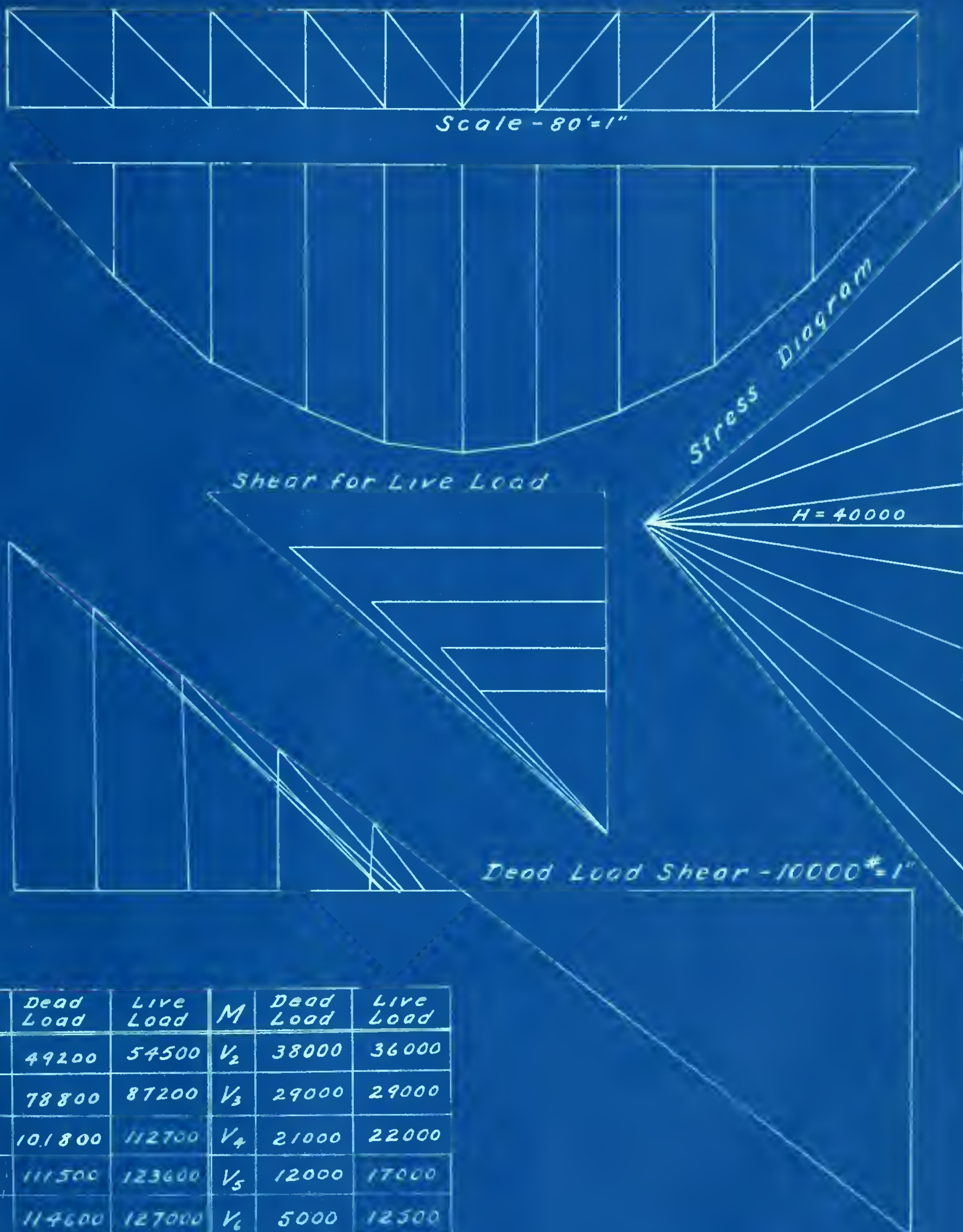
LOWER LATERAL SYSTEM



M	Dead Load	Live Load	M	Dead Load	Live Load
U_1	49200	54500	V_2	38000	36000
U_2	78800	87200	V_3	29000	29000
U_3	101800	112700	V_4	21000	22000
U_4	111500	123600	V_5	12000	17000
U_5	114600	127000	V_6	5000	12500
L_2	49200	54500	D_1	57000	66000
L_3	78800	87200	D_2	43000	53000
L_4	101800	112700	D_3	29500	42000
L_5	111500	123600	D_4	16500	30000
V_1	42000	43500	D_5	5500	23000

DESIGN
OF
207 FT. SPAN - TWO HINGED
SPANDREL BRACED ARCH
CIVIL ENGINEERING DEPARTMENT
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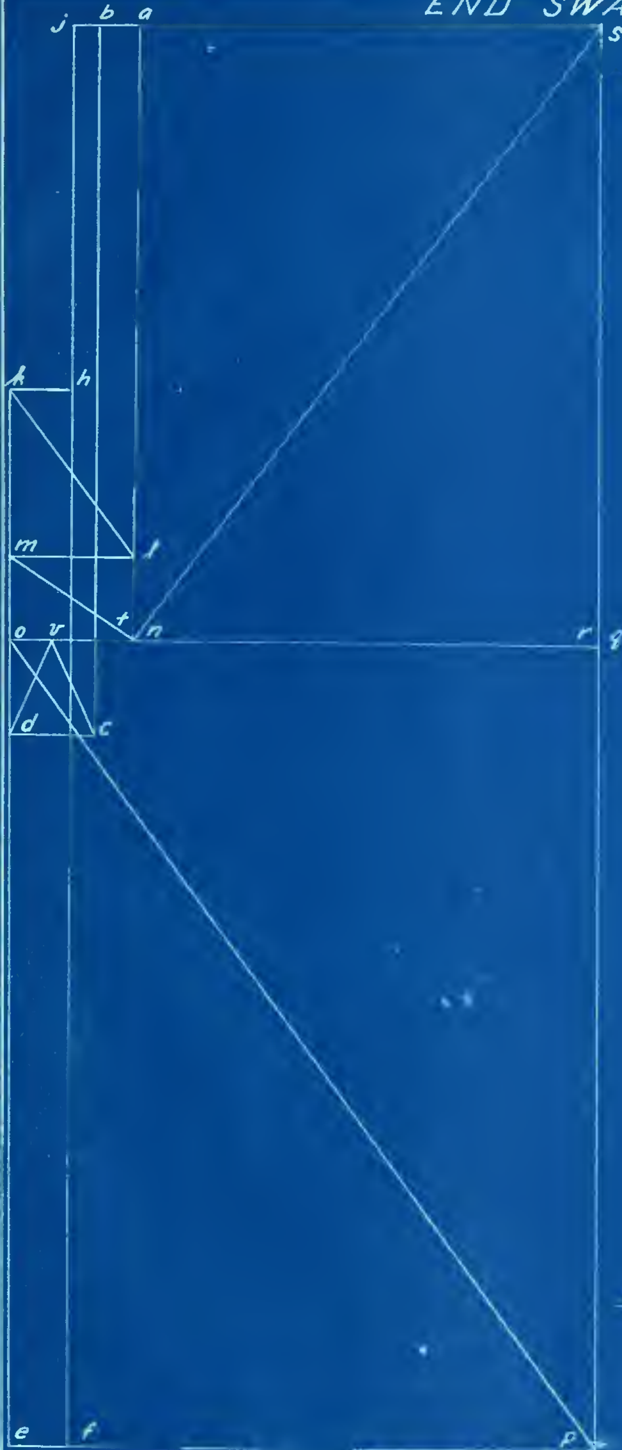
END SWAY BRACING



M	Stress	M	Stress	M	Stress
AL	-84000	QP	-127000	KL	+33000
AN	-97000	PO	+157000	TN	0
AT	-97000	OE	-127000	QN	+71000
BS	-77000	ME	-140000	ON	-21000
ST	+120000	KE	-166000	VR	-84000
SR	-97000	KH	-10000	EP	-93000
RQ	0	MN	+24000	ML	-28000

DESIGN
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207' FT. SPAN - TWO-HINGED
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TABLE II DESIGN OF THE LOWER LATERAL SYSTEM

Members to consist of four angles placed in pairs 25" back to back

Memb.	Equivalent D.L. Stress	Composition of Member	Mom. of Inertia	Area	Rad. of Gyration	Length	Unit Stress	Safe Load	Weight
$L.L.V_1$	107250	4∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{1}{2}$"	163	13.00	3.54	22	8530	110900	980
LLV_2	92000	4∠s 3" x 3" x $\frac{1}{2}$"	136	11.00	3.57	22	8566	94400	830
LLV_3	72500	4∠s 3" x 3" x $\frac{3}{8}$"	105	8.44	3.52	22	8500	71700	630
LLV_4		4∠s 3" x 3" x $\frac{3}{8}$"	105	8.44	3.52	22	8500	71700	630
LLV_5		4∠s 3" x 3" x $\frac{3}{8}$"	105	8.44	3.52	22	8500	71700	630
LLD_1	156000	4∠s 4" x 4" x $\frac{3}{4}$"		9.00			18000	161800	2540
LLD_2	122500	4∠s 4" x 4" x $\frac{9}{16}$"		6.75			18000	121400	1880
LLD_3	92500	4∠s 4" x 4" x $\frac{7}{16}$"		5.25			18000	94500	1420
LLD_4	61500	4∠s 3" x 3" x $\frac{7}{16}$"		3.50			18000	63100	1020
LLD_5		4∠s 3" x 3" x $\frac{3}{8}$"		3.00			18000	54000	670

TABLE III DESIGN OF INTERMEDIATE SWAY BRACING

Member	Inclination to Horizont.	Stress	Net Area	Composition	Weight
SFD_1	55°38'	23920	1.99	1∠ 4" x 3" x $\frac{9}{16}$"	576
SFD_2	42°3'	18180	1.52	1∠ $3\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{5}{8}$"	340
SFD_3	20°38'	16025	1.34	1∠ $3\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{9}{16}$"	270
SFD_4	14°40'	13950	1.16	1∠ $3\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{2}$"	210
SFD_5	10°18'	13720	1.14	1∠ $3\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{2}$"	210

TABLE IV DESIGN OF END SWAY BRACING

Memb.	Stress	Composition of Member	Mom. of Inertia	Area	Rad of Gyration	Length	Unit Stress	Safe Load	Weight
ESH1	109800	2∠s 6" x 6" x $\frac{3}{4}$"	56	16.88	1.83	22.00	7640	129000	1260
ESH2	113000	2∠s 6" x 6" x $\frac{3}{4}$"		13.88		22.00	10000	138800	1260
ESH3	20300	4∠s 3" x 3" x $\frac{3}{8}$"	50	8.44	2.44	22.00	6520	5500	630
$ESD1_E$	151800	2∠s 8" x 8" x $\frac{3}{4}$"	139	22.88	2.47	10.26	7760	177000	800
$ESD1_M$	9600	1∠ 3" x 3" x $\frac{3}{8}$"		.75		10.97	12500	9500	80
ESD2	25000	1∠ 4" x 4" x $\frac{1}{2}$"		2.75		26.22	12500	34400	350
ESD3	33000	1∠ 4" x 4" x $\frac{1}{2}$"		2.75		35.73	12500	34400	460
ESV1	117000	2∠s 8" x 8" x $\frac{1}{2}$"	97	15.50	2.50	7.50	7560	117000	400

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Plate V Design of Floor System

Track Stringers

Span 20.7' Depth 33"

Max. end shear 337500 in.-pounds for live load

$$\frac{450 \times 1}{2 \times 8} \times 20.8^2 \times 12 = 144500 \text{ in-lbs D.L.}$$

Total L.L. moment = 3447250 in.-lbs

Use $6'' \times 6'' \times \frac{11}{16}''$ $\angle$s $\bar{X} = \frac{7.78 + 1.75 - .6875 \times .875(2.5 + 4.75 + 3.44)}{7.78 - 3 \times .6875 \times .875}$

$\bar{X} = \frac{13.62 - 4.62}{7.78 - 1.80} = 1.80$ in. Effective depth = 33 - 1.80 - 1.75 = 29.45"

Unit stress = $\frac{3,447250}{29.45 \times 2 \times 5.98}$ = 9790 #/sq.in.

End Shear = 63900#

Allowed Stress = 10,000 - 75 H $H = \frac{d}{t}$

$63900 = (10000 - 75\frac{d}{t})\,25t$

$= 250000t - 55,200$ $t = \frac{63900 + 55200}{250000} = \frac{1}{2}''$

Weight 2190 + 1160 = 3350#

Stringer Sway Bracing

Length, middle, 8.46 Length, ends, 7.52 Sec θ_1 = 1.058 θ_2 1.074

Load 9300 #/panel Stress, middle, 9300 × 1.058 = 9860, end = 9300 × 1.074

Area, end, .82 sq.in 1 $\angle$ $3'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$ wt. = 57

Area, mid. .83 sq.in. 1 $\angle$ $3'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$ wt. = 92

Area, upper 3.56 sq.in 2 $\angle$s $3'' \times 3'' \times \frac{5}{8}''$ wt. = 98

Floor Beam

Span 22' Depth 40" D.L. mom. = 2×3350 + 450×20.7 = 15700#

L.L. Floor beam reaction 175000# Equiv. total L.L. 182600

Max Moment = 91400 × 7 × 12 = 7700000

Use $8'' \times 8'' \times \frac{7}{8}''$ $\angle$s $\bar{X} = \frac{13.23 \times 2.32 - 6.75(3 + 6 + .875)}{13.23 - 4 \times .875} = \frac{30.72 - 8.64}{13.23 - 3.50} = 2.27$

Effective depth = 39.41" Unit Stress = $\frac{7700000}{2 \times 9.73 \times 39.41}$ = 10020 #/sq.in

End Shear = 2 × 63900 = 127800# Allowed Stress = 10000 - 75 H

$S = (10000 - 75\frac{d}{t})\,28t = 280000t - 21000$ $t = \frac{127800 + 82800}{280000} = \frac{3}{4}''$

Weight = 3960 + 2240 = 6200#

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TABLE V. CONSTANTS OF THE TRUSS MEMBERS

Memb.	Length	u	V	$\frac{u}{V}$	y	$T=\frac{y}{V}$	TL	$\frac{u}{V}TL$	$\frac{p}{V}$	$\frac{p}{V}TL$	T^2L
U_1	20.700	20.700	36.160	.5725	15.84	.4381	9.066	5.196	.5724	5.19	3.97
U_2	20.700	41.400	23.840	1.7365	28.16	1.1812	24.455	42.453	.8683	21.23	28.88
U_3	20.700	62.100	15.040	4.1290	36.96	2.4574	50.863	201.01	1.3763	70.01	124.99
U_4	20.700	82.800	9.760	8.4839	42.24	4.3280	89.587	759.98	2.1209	190.02	387.73
U_5	20.700	103.500	8.000	12.9380	44.00	5.5000	113.840	1472.80	2.5875	294.56	626.10
L_1	26.066	0	41.295		52.00	1.2593	32.835		.5013	16.46	41.33
L_2	24.088	20.700	31.074	.6662	52.00	1.6734	40.310	26.850	.6662	26.84	67.45
L_3	22.493	41.400	21.940	1.8871	52.00	2.3701	53.310	100.59	.9435	50.13	126.35
L_4	21.314	62.100	14.573	4.2618	52.00	3.5682	76.050	323.90	1.4208	108.05	271.36
L_5	20.747	82.800	9.738	8.5035	52.00	5.3400	110.780	942.01	2.1258	235.48	591.55
V_0	52.000	67.953	67.953	1.0000	52.00	.7652	39.789	39.788	.3046	12.12	30.44
V_1	36.160	81.456	60.756	1.3408	52.00	.8559	30.948	41.492	.3407	10.54	26.49
V_2	23.840	97.479	56.079	1.7383	52.00	.9273	22.108	38.425	.3691	8.16	20.52
V_3	15.040	121.064	58.964	2.0531	52.00	.8819	13.265	27.232	.3511	4.66	11.70
V_4	9.760	197.592	114.792	1.7230	52.00	.4530	4.421	7.616	.1803	.80	2.00
V_5	8.000										
D_1	41.666	67.953	58.974	1.1522	52.00	.8817	36.732	42.325	.3510	12.89	32.39
D_2	31.573	81.456	45.877	1.7754	52.00	1.1335	35.735	63.532	.4512	16.12	40.51
D_3	25.587	97.479	32.961	2.9576	52.00	1.5776	40.363	119.38	.6280	25.35	63.69
D_4	22.885	121.064	24.573	4.9265	52.00	2.1161	48.430	238.58	.8424	40.80	102.48
D_5	22.141	197.592	41.381	4.7755	52.00	1.2566	27.823	132.87	.5002	13.92	34.961

TABLE VI $\frac{10-n}{10}\frac{u}{V}$

$n=$	1	2	3	4	5	6	7	8	9
$\frac{10-n}{10}$	.9	.8	:7	.6	.5	.4	.3	.2	.1
U_1	.5152	.4580	.4007	.3435	.2862	.2290	.1717	.1145	.0573
U_2	1.5629	1.3893	1.2156	1.0419	.8683	.6946	.5210	.3473	.1736
U_3	3.7160	3.3031	2.8903	2.4773	2.0645	1.6516	1.2384	.8258	.4129
U_4	7.6352	6.7868	5.9383	5.0902	4.2418	3.3915	2.5452	1.6968	.8484
U_5	11.6438	10.3500	9.0563	7.7625	6.4687	5.1750	3.8813	2.5875	1.2935
L_1									
L_2	.5995	.5329	.4664	.3997	.3331	.2665	.1998	.1332	.0666
L_3	1.6982	1.5096	1.3209	1.1322	.9434	.7548	.5661	.3774	.1887
L_4	3.8355	3.4094	2.9829	2.5570	2.1308	1.7045	1.2783	.8522	.4261
L_5	7.6530	6.8025	5.9520	5.1020	4.2515	3.4012	2.5510	1.7005	.8504
V_0	.9000	.8000	.7000	.6000	.5000	.4000	.3000	.2000	.1000
V_1	1.2041	1.0726	.9386	.8045	.6704	.5363	.4022	.2682	.1341
V_2	1.5646	1.3907	1.2169	1.0430	.8692	.6954	.5215	.3477	.1738
V_3	1.8480	1.6427	1.4374	1.2323	1.0267	.8214	.6160	.4107	.2053
V_4	1.5506	1.3784	1.2061	1.0337	.8615	.6892	.5169	.3446	.1723
V_5	0	0	0	0	1.0000	0	0	0	0
D_1	1.0370	.9218	.8066	.6915	.5761	.4609	.3457	.2304	.1152
D_2	1.5979	1.4203	1.2427	1.0653	.8876	.7101	.5326	.3551	.1775
D_3	2.6619	2.3661	2.0704	1.7746	1.4788	1.1831	.8873	.5915	.2958
D_4	4.4343	3.9416	3.4490	2.9563	2.4636	1.9709	1.4781	.9854	.4927
D_5	4.2976	3.8203	3.3428	2.8652	2.3878	1.9102	1.4328	.9551	.4775

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Table VII $\frac{u-np}{v}$

Mem.	Load	$u-np$	v	$\frac{u-np}{v}$	Memb.	Load	$u-np$	v	$\frac{u-np}{v}$
U_2	1	20.7	23.84	.869	V_2	1	76.779	56.079	1.369
U_3	1	41.4	15.04	2.758	V_3	1	100.364	58.964	1.702
U_3	2	20.7	15.04	1.376	V_3	2	79.664	58.964	1.351
U_4	1	62.1	9.76	6.362	V_4	1	176.892	114.792	1.541
U_4	2	41.4	9.76	4.242	V_4	2	156.192	114.792	1.361
U_4	3	20.7	9.76	2.121	V_4	3	135.492	114.792	1.180
U_5	1	82.8	8.00	10.350	D_2	1	60.756	45.877	1.324
U_5	2	62.1	8.00	7.762	D_3	1	76.779	32.961	2.329
U_5	3	41.4	8.00	5.175	D_3	2	56.079	32.961	1.701
U_5	4	20.7	8.00	2.588	D_4	1	100.364	24.573	4.083
L_3	1	20.7	21.940	.944	D_4	2	79.664	24.573	3.245
L_4	1	41.4	14.573	2.841	D_4	3	58.964	24.573	2.402
L_4	2	20.7	14.573	1.420	D_5	1	176.892	41.381	4.275
L_5	1	62.1	9.738	6.377	D_5	2	156.192	41.381	3.775
L_5	2	41.4	9.738	4.2515	D_5	3	135.492	41.381	3.275
L_5	3	20.7	9.738	2.126	D_5	4	114.792	41.381	2.774

Table VIII. Stresses Due to Vertical Reactions from Unit Loads

n	1	2	3	4	5	6	7	8	9
U_1	.5152	.4580	.4007	.3435	.2862	.2290	.1717	.1145	.0573
U_2	.6939	1.3893	1.2156	1.0419	.8683	.6946	.5210	.3473	.1736
U_3	.9603	1.9268	2.8903	2.4773	2.0645	1.6516	1.2384	.8258	.4129
U_4	1.2737	2.5448	3.8174	5.0902	4.2418	3.3935	2.5452	1.6968	.8484
U_5	1.2936	2.5877	3.8810	5.1748	6.4687	5.1750	3.8813	2.5875	1.2935
L_1	0	0	0	0	0	0	0	0	0
L_2	.5995	.5329	.4664	.3997	.3331	.2665	.1998	.1332	.0666
L_3	.7547	1.5096	1.3209	1.1322	.9435	.7548	.5661	.3774	.1887
L_4	.9945	1.9890	2.9829	2.5570	2.1308	1.7045	1.2783	.8522	.4261
L_5	1.2758	2.5510	3.8263	5.1020	4.2515	3.4012	2.5510	1.7005	.8504
V_0	.9000	.8000	.7000	.6000	.5000	.4000	.3000	.2000	.1000
V_1	1.2041	1.0726	.9386	.8045	.6704	.5363	.4402	.2682	.1341
V_2	.1955	1.3907	1.2169	1.0430	.8692	.6954	.5215	.3477	.1738
V_3	.1455	.2915	1.4374	1.2323	1.0267	.8214	.6160	.4107	.2053
V_4	.0096	.0178	.0258	1.0337	.8615	.6892	.5169	.3446	.1723
V_5	0	0	0	0	1.0000	0	0	0	0
D_1	1.0370	.9216	.8066	.6915	.5761	.4609	.3457	.2304	.1152
D_2	.2737	1.4203	1.2427	1.0653	.8876	.7102	.5326	.3551	.1775
D_3	.3324	.6647	2.0704	1.7746	1.4788	1.1831	.8873	.5915	.2958
D_4	.3513	.6963	1.0740	2.9563	2.4636	1.9709	1.4781	.9854	.4927
D_5	.0226	.0453	.0671	.0907	2.3678	1.9102	1.4328	.9551	.4775

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Table IX. $\Sigma \frac{u}{v} TL + \Sigma n \frac{\rho}{v} TL = \Sigma_1$

$n=$	1	2	3	4	5-9
U_1	5.20	5.20	5.20	5.20	5.20
U_2	21.23	42.45	42.45	42.45	42.45
U_3	70.01	140.02	201.01	201.01	201.01
U_4	190.02	380.04	570.06	759.98	759.98
U_5	294.58	599.16	883.44	1178.32	1472.80
L_1	0	0	0	0	0
L_2	26.88	26.85	26.85	26.85	26.85
L_3	50.13	100.26	100.59	100.59	100.59
L_4	108.05	216.10	324.15	323.90	323.90
L_5	235.48	470.96	706.44	941.92	942.01
V_0	39.79	39.79	39.79	39.79	39.79
V_1	41.49	41.49	41.49	41.49	41.49
V_2	8.16	38.42	38.42	38.42	38.42
V_3	4.66	9.31	27.23	27.23	27.23
V_4	.80	1.59	2.39	7.62	7.62
V_5	0	0	0	0	0
D_1	42.33	42.33	42.33	42.33	42.33
D_2	16.12	63.53	63.53	63.53	63.53
D_3	25.35	50.69	119.38	119.38	119.38
D_4	40.79	81.59	122.38	238.58	238.58
D_5	13.92	27.63	41.75	55.67	132.87
Σ_1	1234.99	2377.61	3399.18	4254.26	4626.03
Σ_1/Σ_2	.2108	.4058	.5802	.7262	.7896

Table X $\left(\frac{\Sigma \frac{u}{v} TL + \Sigma n \frac{\rho}{v} TL}{2 \Sigma T^2 L}\right) T$

U_1	.0923	.1778	.2542	.3184	.3460
U_2	.2490	.7353	.6853	.8584	.9327
U_3	.5180	.9971	1.4255	1.7856	1.9404
U_4	.9123	1.7560	2.5109	3.1451	3.4175
U_5	1.1593	2.2318	3.1909	3.9967	4.3455
L_1	.2654	.5110	.7306	.9151	.9943
L_2	.3528	.6790	.9709	1.2160	1.3214
L_3	.4996	.9618	1.3751	1.7224	1.8715
L_4	.7522	1.4470	2.0703	2.5930	2.6175
L_5	1.1256	2.1669	3.0981	3.8805	4.2165
V_0	.1613	.3105	.4349	.5560	.6042
V_1	.1804	.3474	.4965	.6220	.6758
V_2	.1955	.3763	.5380	.6739	.7322
V_3	.1859	.3579	.5115	.6409	.6964
V_4	.0955	.1838	.2626	.3161	.3577
V_5	0	0	0	0	0
D_1	.1859	.3578	.5114	.6407	.6962
D_2	.2390	.4600	.6576	.8237	.8950
D_3	.3325	.6402	.9153	1.4465	1.2456
D_4	.4461	.8583	1.2278	1.5378	1.6789
D_5	.2649	.5098	.7291	.9132	.9922

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Table XI Stresses due to Unit Panel Loads

n	1	2	3	4	5	6	7	8	9
U_1	-.4229	-.2802	-.1455	-.0251	+.0598	+.1170	+.1743	+.2315	+.2817
U_2	-.4449	-.6540	-.5303	-.1635	+.0644	+.2381	+.4117	+.5854	+.7591
U_3	-.4423	-.9297	-1.4648	-.6917	-.1241	+.2888	+.7020	+1.1146	+1.5275
U_4	-.3614	-.7888	-1.3065	-1.9451	-.8243	+.0240	+.8723	+1.7207	+2.5691
U_5	-.1345	-.3559	-.6901	-1.1781	-2.1232	-.8295	+.4642	+1.7580	+3.0520
L_1	-.2654	-.5110	-.7306	-.9151	-.9943	-.9943	-.9943	-.9943	-.9943
L_2	+.2467	-.1461	-.5045	-.8163	-.9983	-1.0549	-1.2116	-1.1882	-1.2548
L_3	+.2551	+.5478	-.0542	-.5902	-.9280	-1.1177	-1.3054	-1.4941	-1.6828
L_4	+.2[illegible]23	+.5420	+.9126	-.0360	-.6867	-1.1130	-.5392	-1.9653	-2.3914
L_5	+.1502	+.3841	+.7252	+1.2215	+.0350	-.8153	-1.6653	-2.5170	-3.3661
V_0	-.7387	-.4895	-.2651	-.0440	+.1042	+.2042	+.3042	+.4042	+.5042
V_1	-1.0237	-.7252	-.4421	-.1625	+.0054	+.1395	+.2356	+.4076	+.5417
V_2	0	-1.0144	-.6789	-.3691	-1.370	+.0368	+.2107	+.3845	+.5584
V_3	+.0401	+.0664	-.9259	-.5916	-.3303	-.1250	-.0804	+.2857	+.4911
V_4	+.0859	+.1660	-.2370	-.7176	-.5038	-.3315	-.1952	+.0131	+.1854
V_5	0	0	0	0	-1.0000	0	0	0	0
D_1	+.8511	+.5640	+.2957	+.0508	-.1201	-.2353	-.3505	-.4658	-.5810
D_2	+.0347	+.9603	+.5851	+.2416	-.0074	-.1849	-.3524	-.5399	-.7175
D_3	0	+.0245	+1.1551	+.3281	+.2332	-.0625	-.3583	-.6541	-.9498
D_4	-.0948	-.1620	-.1538	+1.4155	+.7647	+.2920	-.2006	-.6935	-1.1862
D_5	-.2423	-.4646	-.6620	-.8225	+1.3956	+.9180	+.4402	-.0371	-.5147

Table XII Summary of Stresses from Unit Panel Loads

n	1+9	2+8	3+7	4+6	5	Total +	Total −	Excess +	Excess −
U_1	-.1342	-.0487	-.0288	-.0919	-.0598	.8713	.8737	.4630	.5684
U_2	+.3142	-.0686	-.1186	+.0546	+.0644	2.0587	1.8127	1.1708	.9752
U_3	+1.0852	+.1859	+.7628	-.4029	-.1241	3.6329	3.6526	2.2295	1.9071
U_4	+2.2087	+.9319	-.4342	-1.9211	-.8243	5.1661	4.2261	3.6414	2.7339
U_5	2.9175	+1.4021	-.2259	-2.0076	-2.1232	5.2742	5.3113	3.5162	2.8133
L_1	-1.2597	-1.5053	-1.7249	-1.9094	-.9943	0	7.3936	0	1.9886
L_2	-1.0081	-1.3343	-1.6261	-1.8712	-.9883	.2467	7.0747	.2467	2.3764
L_3	-1.4277	-.9463	-1.3596	-1.7079	-.9280	.8029	7.1724	.5478	2.9882
L_4	-2.1591	-1.4233	-.6266	-1.1490	.6867	1.6969	7.3715	1.1549	3.9306
L_5	-3.2159	-2.1329	-.9373	+.4062	+.0352	2.5190	8.3639	1.6056	5.0316
V_0	-.2345	-.0853	+.0381	+.1602	+.1042	1.5210	1.5373	.8084	1.0038
V_1	-.4820	-.3176	-.2069	-.0430	+.0054	1.3298	2.3735	.7773	1.4658
V_2	+.5584	-.6299	-.4682	-.3323	-.1370	1.1904	2.1994	.7691	1.3835
V_3	+.5312	+.3521	+.8455	-.7166	-.3303	.9637	1.9728	.5715	1.2562
V_4	+.2713	+.1791	-.3962	-1.0491	-.5038	.4504	1.9491	.1854	1.0491
V_5	0	0	0	0	-1.0000	0	1.0000	0	1.0000
D_1	+.2701	+.0982	-.0553	-.1845	-.1201	1.7611	1.7527	1.1463	.9315
D_2	-.6828	+.4204	+.2327	+.0557	-.0074	1.8217	1.8021	1.2019	1.0689
D_3	-.9498	-.6296	+.8068	+.2656	+.2332	1.7409	2.0247	1.3883	1.3081
D_4	-1.2810	-.8555	-.3546	+1.7105	+.7647	2.4952	2.4911	1.7105	1.3870
D_5	-.7370	-.5017	-.2218	+.8055	+1.3956	2.7538	2.7432	1.8358	1.2871

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TABLE XIII. STRESSES DUE TO DEAD LOAD

n=	1+9	2+8	3+7	4+6	5	Total +	Total −
P=	24.3	21.4	19.9	18.7	17.3		
U_1	−3.26	−1.04	+.56	+1.72	+1.04	3.32	4.30
U_2	+7.66	−6.95	−2.34	+1.05	+1.13	9.84	9.29
U_3	+26.32	+4.00	−15.13	−7.53	−2.12	30.32	24.78
U_4	+53.71	+20.02	−8.58	−36.89	−14.20	73.73	59.67
U_5	+70.94	+30.05	−4.46	−37.58	−36.71	100.99	78.75
L_1	−30.64	−32.23	−34.35	−35.71	−17.22		150.35
L_2	−24.53	−28.59	−32.35	−35.00	−17.12		137.62
L_3	−34.73	−20.28	−27.10	−31.92	−16.09		130.02
L_4	−52.28	−30.44	−12.53	−21.49	−11.93		128.67
L_5	−78.25	−48.71	−18.76	+6.59	+.53		137.60
V_0	−5.68	−1.82	+.95	+2.99	+1.81	5.75	7.50
V_1	−11.70	−6.76	−4.10	−.80	−.10		23.48
V_2	+13.59	−13.46	−9.30	−6.20	−2.36	13.59	34.32
V_3	+12.93	+7.55	−16.81	−13.39	−5.70	20.48	35.90
V_4	+6.60	+3.84	+1.56	−20.37	−8.71	12.00	29.08
V_5	0	0	0	0	−17.30	0	17.30
D_1	+6.54	+2.09	−1.12	−3.45	−2.09	8.63	6.66
D_2	−16.82	+8.96	+4.42	+1.06	+.10	14.54	16.82
D_3	−23.11	−13.50	+15.84	+10.56	+4.01	30.47	36.61
D_4	−30.93	−18.19	−6.92	+32.12	+13.68	45.80	56.04
D_5	−18.42	−10.75	−4.41	+1.79	+24.13	25.92	33.58

TABLE XIV. STRESSES DUE TO WIND LOADS

n=	1	2	3	4	5	6	7	8	9
P=	60.2	35.7	19.0	10.0	7.4	10.1	19.0	35.7	60.2
U_1	−25.47	−10.00	−2.78	−.26	+.44	+1.19	+3.32	+8.27	+17.34
U_2	−26.79	−32.49	−10.07	−1.86	+.49	+2.41	+7.84	+20.93	+45.75
U_3	−26.63	−33.18	−27.82	−7.00	−.91	+2.93	+13.37	+39.86	+92.07
U_4	−21.97	−28.17	−24.81	−19.63	−6.07	+7.31	+16.63	+61.55	+154.88
U_5	−8.11	−12.72	−13.10	−11.93	−15.71	−8.36	+9.15	+62.82	+183.82
L_1	−15.98	−18.24	−13.88	−9.23	−7.36	−10.05	−18.91	−35.57	−59.91
L_2	+14.86	−5.52	−9.58	−8.23	−7.33	−10.67	−21.33	−42.68	−75.61
L_3	+15.36	+19.57	−1.03	−5.95	−6.88	−11.30	−24.83	−53.41	−101.41
L_4	+14.60	+19.31	+17.35	−.34	−5.10	−11.26	−29.29	−70.26	−144.13
L_5	+9.05	+13.72	+13.95	12.36	+.23	−8.28	−31.72	−89.97	−202.87
V_0	−44.47	−17.48	−4.87	−.45	+.77	+2.07	+5.79	+14.45	+30.39
V_1	−61.62	−25.89	−8.40	−1.84	+.05	+1.41	+4.49	+14.58	+32.65
V_2	0	−36.26	−12.90	−3.73	−1.01	+.38	+4.01	+13.76	+33.76
V_3	+2.42	+2.36	−17.59	−5.98	−2.44	−1.26	+1.54	+10.23	+29.30
V_4	+5.17	+5.96	+4.45	−7.12	−3.73	−3.34	−3.02	+.48	+11.18
V_5	0	0	0	0	−7.40	0	0	0	0
D_1	+51.22	+20.14	+5.61	+.49	−.90	−2.39	−6.67	−16.66	−35.01
D_2	+2.09	+34.27	+11.11	+2.46	−.06	−1.88	−6.90	−19.32	−43.25
D_3	0	+.88	+21.95	+6.35	+11.71	−.64	−6.83	−23.40	−57.24
D_4	−5.70	−5.78	−2.91	+14.34	+5.85	+3.03	−3.69	−24.51	−71.02
D_5	−14.58	−16.58	−12.57	−6.29	+10.32	+9.36	+8.26	−1.35	−31.05

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Table XV. Total Maximum and Minimum Stresses

Member	Dead L'd	Wind Load		Uniform Live L		Excess Live Load		Total Stress	
	Max+, Min-	Max ±	Min ∓	Max +	Min -	Max. +	Min. -	Max. +	Min -
U_1	-.98	30.56	38.51	69.38	69.58	15.31	18.79	122.71	127.37
U_2	+.55	77.42	71.21	163.77	164.57	38.66	36.11	280.13	277.82
U_3	+5.64	148.23	95.64	289.49	290.32	73.68	62.92	514.22	498.65
U_4	+14.06	240.37	100.65	413.43	415.36	113.81	90.24	774.64	738.94
U_5	+22.64	255.79	69.87	419.77	422.25	116.16	92.81	802.84	759.73
L_1	-150.35	0	189.13	0	588.08	0	65.70	113.95	918.08
L_2	-137.62	14.86	180.65	19.62	562.85	8.14	78.51	139.60	890.82
L_3	-130.02	30.93	204.81	68.32	570.69	18.08	98.75	221.70	939.26
L_4	-128.67	51.26	260.38	134.80	615.60	38.12	129.82	368.97	1070.14
L_5	-137.60	49.30	332.84	199.40	666.24	53.09	166.32	516.53	1234.20
V_0	-32.35	53.47	158.17	121.65	230.44	26.72	49.15	290.37	453.94
V_1	-23.48	53.18	97.75	105.96	188.68	25.69	47.37	217.66	345.54
V_2	-17.73	51.81	53.90	94.92	175.55	25.33	45.67	165.34	283.94
V_3	-15.42	45.85	27.27	76.85	156.78	18.89	41.44	133.68	251.78
V_4	-16.08	27.24	17.21	114.71	135.00	6.13	34.18	140.04	204.46
V_5	-17.30	0	7.40	0	79.50	0	33.00	-1.15	128.55
D_1	+1.97	77.46	61.63	140.03	138.54	36.83	30.79	255.70	245.89
D_2	-2.08	49.93	71.41	144.87	154.70	39.37	35.71	254.61	262.86
D_3	-6.20	30.89	88.11	162.25	161.41	45.77	43.26	293.03	295.88
D_4	-10.24	23.22	113.61	199.49	196.53	56.74	45.30	364.72	360.56
D_5	-7.66	27.94	84.42	218.67	218.17	60.50	42.43	359.76	348.85

Table XVI. Preliminary Design of Members

Member	Effective Stress	Unit Stress	Area
U_1	270640	11100	39.56
U_2	602300	11100	54.09
U_3	1092020	11100	99.94
U_4	1630350	11100	149.44
U_5	1682400	11100	155.44
L_1	1211090	11320	106.74
L_2	1202000	11320	106.74
L_3	1340060	11330	120.24
L_4	1638380	11550	141.22
L_5	1976900	11550	172.92
V_0	823490	8750	96.00
V_1	623560	8760	71.25
V_2	499450	8600	57.62
V_3	430660	9940	48.25
V_4	379790	10000	37.00
V_5	128550	8100	17.64
D_1	540540	13000	61.16
D_2	559860	13000	63.06
D_3	636360	13000	75.00
D_4	782810	13000	89.00
D_5	763990	13000	85.00

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TABLE XVII. DEAD LOAD ON ONE TRUSS

Panel Point	0	1	2	3	4	5
1 Truss	27650	29300	26340	28290	29940	30160
½ Floorbeam		2595	2595	2595	2595	2595
Stringer	10900	3060	3060	3060	3060	3060
Track	6826	4658	4658	4658	4658	4658
Upper Lat	117	233	233	233	233	233
Stringer Brac	110	220	220	220	220	220
Sway Frames	6100	578	578	271	213	210
Lower Lat.	1760	2625	1965	1535	1255	755
Total	53470	43269	39412	40862	42174	39891
+20%	10694	8654	7882	8172	8435	7968
Total D.L	64164	51923	47294	49034	50609	47869

TABLE XVIII. CONSTANTS OF THE TRUSS

Member	$\frac{u}{v}\frac{TL}{A}$	$\frac{p}{v}\frac{TL}{A}$	$\frac{T^2L}{A}$	Temperature Stress -25°F	Temperature Stress +125°F
U_1	.1299	.1298	.0993	-3.12	-2.07
U_2	.7860	.3932	.5349	-8.41	+5.61
U_3	2.1001	.7001	1.2499	-17.47	+11.65
U_4	5.1005	1.2754	2.6025	-23.70	+15.80
U_5	9.5030	1.9006	4.0400	-39.15	+26.12
L_1	0	.1538	.3863	+8.35	-5.56
L_2	.2509	.2506	.6304	+11.91	-7.95
L_3	.8363	.4177	1.0529	+16.86	-11.24
L_4	2.2973	.7664	1.9245	+26.12	-17.40
L_5	5.4445	1.3612	3.4192	+38.00	-25.15
V_0	.4145	.1264	.3171	-5.45	+3.63
V_1	.5763	.1466	.3679	-6.09	+4.06
V_2	.6623	.1407	.3538	-6.60	+4.40
V_3	.5674	.0970	.2438	-6.28	+4.19
V_4	.2059	.0215	.0542	-3.23	+2.16
V_5	0	0	0	0	0
D_1	.6940	.2114	.5310	+6.27	-4.19
D_2	1.0084	.2559	.6430	+8.06	-5.18
D_3	1.5918	.3380	.8492	+11.22	-7.48
D_4	2.6209	.4584	1.1515	+15.02	-10.02
D_5	1.5633	.1637	4.1130	+8.95	-5.97
			24.5644		

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TABLE XIX $\Sigma \frac{U}{V} \frac{TL}{A} + \Sigma \frac{D}{V} \frac{TL}{A}$

n	1	2	3	4	5
U_1	.1299	.1299	.1299	.1299	.1299
U_2	.3932	.7860	.7860	.7860	.7860
U_3	.7001	1.4002	2.1001	2.1001	2.1001
U_4	1.2754	2.5508	3.8262	5.1005	5.1005
U_5	1.9006	3.8012	5.7018	7.6024	9.5030
L_1	0	0	0	0	0
L_2	.2508	.2509	.2509	.2509	.2509
L_3	.4177	.8354	.8383	.8383	.8383
L_4	.7664	1.5328	2.2992	2.2973	2.2973
L_5	1.3612	2.7624	4.0836	5.4436	5.4445
V_0	.4145	.4145	.4145	.4145	.4145
V_1	.5763	.5763	.5763	.5763	.5763
V_2	.1407	.6623	.6623	.6623	.6623
V_3	.0970	.1940	.5674	.5674	.5674
V_4	.0215	.0430	.0645	.2059	.2059
V_5	0	0	0	0	0
D_1	.6940	.6940	.6940	.6940	.6940
D_2	.2559	1.0084	1.0084	1.0084	1.0084
D_3	.3380	.6760	1.5918	1.5918	1.5918
D_4	.4584	.9168	1.3752	2.6809	2.6809
D_5	.1637	.3274	.4911	.6948	1.5633
Σ_1	10.3553	19.5623	27.4615	33.6065	36.4153
Σ_1/Σ_2	.2108	.3982	.5590	.6842	.7413

TABLE XX. $\left(\frac{\Sigma \frac{U}{V} \frac{TL}{A} + \Sigma n \frac{D}{V} \frac{TL}{A}}{2 \Sigma \frac{T^2 L}{A}}\right) T$

U_1	+.0923	.1744	.2449	.2997	.3248
U_2	+.2490	.4703	.6603	.8061	.8756
U_3	+.5180	.9784	1.3736	1.6812	1.8217
U_4	+.9123	1.7233	2.4192	2.9617	3.2083
U_5	+1.1593	2.1900	3.0744	3.7626	4.0772
L_1	-.2654	.5014	.7039	.8616	.9334
L_2	-.3528	.6664	.9354	1.1443	1.2405
L_3	-.4996	.9437	1.3249	1.6215	1.7570
L_4	-.7522	1.4209	1.9946	2.4416	2.6453
L_5	-1.1256	2.1263	2.9850	3.6534	3.9536
V_0	+.1613	.3047	.4277	.5235	.5672
V_1	+.1804	.3408	.4785	.5856	.6345
V_2	+.1955	.3692	.5184	.6344	.6874
V_3	+.1659	.3512	.4930	.6034	.6536
V_4	+.0955	.1804	.2532	.3099	.3356
V_5	0	0	0	0	0
D_1	-.1859	.3511	.4929	.6033	.6536
D_2	-.2390	.4513	.6336	.7756	.8403
D_3	-.3325	.5282	.8816	1.0793	1.1693
D_4	-.4461	.8426	1.1830	1.4479	1.5687
D_5	-.2649	.5003	.7024	.8597	.9314

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Table XXI Stresses Due to Unit Panel Loads

n	1	2	3	4	5	6	7	8	9
U_1	-.4229	-.2836	-.1558	-.0438	+.0386	+.0958	+.1531	+.2103	+.2676
U_2	-.4449	-.9190	-.5553	-.2338	+.0073	+.1810	+.3546	+.5283	+.7020
U_3	-.4423	-.9484	-1.5167	-.7961	-.2428	+.1701	+.5833	+.9959	+1.4088
U_4	-.3614	-.8215	-1.3982	-2.1285	-1.0335	-.1852	+.6631	+1.5115	+2.5399
U_5	-.1345	+.3977	-.8065	-1.4120	-2.3915	-1.0978	+1.959	+1.4697	+2.7837
L_1	-.2654	-.5014	-.7039	-.8616	-.9334	-.9334	-.9334	-.9334	-.9334
L_2	+.2467	-.1335	-.4690	-.7451	-.9074	-.9740	-1.0407	-1.1073	-1.1739
L_3	+.2551	+.5659	-.0040	-.4893	-.8135	-1.0022	-1.1909	-1.3796	-1.5683
L_4	+.2423	+.5681	+.9883	+.1158	-.5145	-.9408	-1.3670	-1.7931	-2.2192
L_5	+.1502	+.4247	+.8413	+1.4486	+.2929	-.5574	-1.4076	-2.2581	-3.1012
V_0	-.7387	-.4353	-.2723	-.0765	+.0672	+.1672	+.2672	+.3672	+.4672
V_1	-1.0237	-.7318	-.4601	-.2189	-.0359	+.0982	+.1943	+.3663	+.5004
V_2	0	-1.0215	-.5988	-.4086	-.1818	-.0080	+.1659	+.3397	+.5136
V_3	+.0401	+.0537	-.9444	-.6289	-.3729	-.1676	+.0378	+.2431	+.4485
V_4	+.0859	+.1626	+.2274	-.7238	-.5257	-.3534	-.1811	-.0088	+.1635
V_5	0	0	0	0	-1.0000	0	0	0	0
D_1	+.6511	+.5707	+.3137	+.0882	-.0775	-.1927	-.3079	-.4232	-.5384
D_2	+.0347	+.9690	+.6091	+.2887	+.0473	-.1302	-.3177	-.4852	-.6628
D_3	0	+.0365	+1.1886	+.6953	+.3095	+.0138	-.2820	-.5778	-.8735
D_4	-.0948	-.1453	-1.090	+1.5084	+.8949	+.4022	-.0906	-.5833	-1.0760
D_5	-.2423	-.4550	-.6353	-.7690	+1.4564	+.9788	+.5014	+.0237	-.4539

Table XXII Summary of Stresses Due to Unit Panel Loads

n	1+9	2+8	3+7	4+6	5	Total +	Total −	Excess +	Excess −
U_1	-.1554	-.0733	-.0027	+.0520	+.0386	.7653	.9061	.4206	.5787
U_2	+.2571	-.3907	-.2007	+.0528	+.0073	1.7732	2.1530	1.0560	1.1528
U_3	+.9665	+.0475	-.9334	-.6260	-.2428	3.1581	3.9463	1.9921	1.9590
U_4	+2.0485	+.6900	-.7351	-2.3137	-1.0335	4.5345	5.9283	3.0230	2.9500
U_5	+2.4692	+1.0920	-.6107	-2.5098	-2.3915	4.4693	6.2401	2.9796	3.1981
L_1	-1.1988	-1.4348	-1.6373	-1.7950	-.9334	0	6.9993	0	1.8664
L_2	-.9272	-1.2408	-1.5097	-1.7191	-.9074	.2467	6.5509	.2467	2.2146
L_3	-1.3132	.8137	-1.1949	-1.5915	-.8135	.8210	6.4478	.5659	2.7592
L_4	-1.9769	-1.2250	-.3787	-.8250	-.5145	1.9145	6.8346	1.2306	3.5862
L_5	-2.9380	-1.8334	-.5663	+.8912	+.2929	3.1577	7.3313	1.8733	4.5158
V_0	-.2715	-.1281	-.0051	+.0907	+.0672	1.3360	1.5828	.7344	1.0110
V_1	-.5233	-.3655	-.2658	-.1207	-.0359	1.1592	2.4704	.6947	1.4838
V_2	+.5136	+.6816	-.5326	-.4166	-.1818	1.0192	2.3184	.6795	1.4301
V_3	+.4886	+.3028	+.9066	-.7965	-.3729	.8292	2.1138	.4863	1.3173
V_4	+.2494	+.1538	+.0463	-1.0872	-.5257	.6394	1.7928	.3133	1.0772
V_5	0	0	0	0	-1.0000	0	1.0000	0	1.0000
D_1	+.3127	+.1475	+.0058	-.1045	-.0775	1.8237	1.5397	1.1648	.8463
D_2	-.6281	+.4838	+.2914	+.1595	+.0473	1.9496	1.5959	1.2587	.9805
D_3	-.8735	-.5413	+.9066	+.7091	+.3095	2.2437	1.7333	1.4981	1.1555
D_4	-1.1708	-.7296	-.1996	+1.9016	+.8949	2.8055	2.1000	1.9106	1.1666
D_5	-.6962	-.4213	-.1339	+.2098	+1.4564	2.9603	2.5555	1.9578	1.2240

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TABLE XXIII STRESSES FROM DEAD LOAD

n	1+9	2+8	3+7	4+6	5
P	51.9	47.3	49.0	50.6	47.9
U_1	-8.07	-3.47	-.13	+2.63	+1.85
U_2	+13.34	-18.48	-9.83	+2.67	+.35
U_3	+50.16	+2.25	-45.77	-31.68	-11.63
U_4	+124.99	+32.64	-36.02	-117.06	-49.50
U_5	+137.49	+51.65	-29.92	-127.00	-114.55
L_1	-62.22	-67.87	-80.22	-90.83	-44.71
L_2	-48.12	-58.69	-73.97	-86.98	-43.46
L_3	-68.16	-38.49	-58.54	-80.53	-36.96
L_4	-102.60	-57.94	-18.56	-41.75	-24.64
L_5	-153.52	-86.72	-27.75	+45.10	+14.03
V_0	-14.09	-6.06	-.25	+4.59	+3.22
V_1	-27.16	-17.29	-13.02	-6.11	-1.72
V_2	+26.65	-32.25	-26.09	-21.08	-8.71
V_3	+25.36	+14.32	-44.42	-40.30	-17.86
V_4	+12.94	+7.27	+2.27	-55.01	-25.18
V_5	0	0	0	0	-47.90
D_1	+16.23	+6.98	+.25	-5.29	-3.71
D_2	-32.60	+22.88	+14.28	+8.07	+2.26
D_3	-45.37	-25.60	+44.42	+35.88	+11.62
D_4	-60.76	-34.51	-9.78	+96.67	+42.87
D_5	-36.16	-19.97	-6.56	+10.62	+69.76

TABLE XXIV. STRESSES FROM D.W.L.

n	1+9	2+8	3+7	4+6	5
P	23.3	13.0	5.9	2.2	1.5
U_1	-3.62	-.95	-.02	+.11	+.06
U_2	+5.99	-5.08	-1.18	+.12	+.01
U_3	+22.52	+.62	-5.50	-1.38	-.36
U_4	+47.74	+8.97	-4.33	-5.09	-1.55
U_5	+61.72	+14.20	-3.60	-5.52	-3.59
L_1	-27.92	-18.65	-9.66	-3.95	-1.40
L_2	-21.60	-16.13	-8.90	-3.78	-1.36
L_3	-30.60	-10.58	-7.05	-3.50	-1.22
L_4	-46.06	-15.92	-2.23	-1.82	-.77
L_5	-68.92	-23.82	-3.34	+1.96	+.44
V_0	-6.32	-1.67	-.03	+.20	+.10
V_1	-12.19	-4.75	-1.57	-.27	-.05
V_2	+11.97	-8.66	-3.14	-.92	-.27
V_3	+11.38	+3.94	-5.35	-1.75	-.56
V_4	+5.81	+2.00	+.27	-2.39	-.79
V_5	0	0	0	0	-1.50
D_1	+7.31	+1.92	+.03	-.23	-.12
D_2	-14.63	+6.29	+1.72	+.35	+.07
D_3	-20.35	-7.04	+5.35	+1.56	+.46
D_4	-27.28	-9.48	-1.18	+4.20	+1.34
D_5	-16.22	-5.48	-.79	+4.62	+2.18

TABLE XXV. STRESSES FROM LIVE WIND LOADS

n	1	2	3	4	5	6	7	8	9
P	36.8	22.8	13.1	7.9	5.9	7.9	13.1	22.8	36.8
U_1	-15.55	-6.47	-2.04	-.35	+.23	+.76	+2.01	+4.79	+9.84
U_2	-16.36	-20.35	-7.28	-1.85	+.04	+1.43	+4.65	+12.04	+25.83
U_3	-16.28	-21.63	-19.87	-6.29	-1.43	+1.34	+7.65	+22.71	+51.84
U_4	-13.30	-18.73	-18.33	-16.88	-6.10	-1.46	+8.70	+34.46	+86.84
U_5	-4.45	-9.08	-10.56	-11.13	-14.11	-8.68	+2.67	+33.97	+102.43
L_1	-9.78	-11.42	-9.23	-6.81	-5.51	-7.38	-12.45	-21.28	-34.35
L_2	+9.08	-3.04	-6.15	-5.89	-5.45	-7.70	-13.64	-25.27	-43.20
L_3	+9.40	+12.89	-.05	-3.87	-4.80	-7.91	-15.61	-31.45	-57.71
L_4	+8.93	+12.96	+12.95	+.92	-3.04	-7.43	-17.92	-40.89	-81.68
L_5	+5.53	+9.69	+11.02	+11.43	+1.73	-4.40	-18.46	-51.48	-114.38
V_0	-27.20	-11.29	-3.57	-.60	+.40	+1.32	+3.51	+8.38	+17.19
V_1	-37.75	-16.67	-6.03	-1.73	-.21	+.78	+2.55	+8.36	+18.41
V_2	0	-23.27	-9.17	-3.23	-1.07	-.06	+2.18	+7.76	+18.90
V_3	+1.47	+1.36	-12.37	-4.97	-2.20	-1.32	+.50	+5.54	+16.50
V_4	+3.16	+3.71	+2.98	-5.72	-3.10	-2.79	-2.38	-.20	+6.01
V_5	0	0	0	0	5.90	0	0	0	0
D_1	+31.30	+13.01	+4.12	+.70	-.46	-1.52	-4.04	-9.65	-19.81
D_2	+1.28	+22.10	+7.99	+2.29	+.28	-1.03	-4.17	-11.06	-24.39
D_3	0	+.83	+15.57	+5.49	+1.83	+.11	-3.70	-13.16	-32.1
D_4	-3.49	-3.34	-1.43	+11.90	+5.28	+3.18	-1.19	-13.29	-39.5
D_5	-8.92	-10.37	-8.33	-6.08	+8.59	+7.73	+6.67	+.55	-16.70

TABLE XXVI MAXIMUM AND MINIMUM STRESSES

Member	Dead Load	Dead Wind Load		Live Wind Loads				Excess Live Load		Uniform Live Load		Temperature		Total Stress	
		Overturn'g	Low. Lat.	OVERTURNING		LOWER LATERAL									
				Max ±	Min. ∓	Max. ±	Min ∓	Max. +	Min. −	Max. +	Min −	Max. +	Min −	Max +	Min. −
U_1	-7.19	-4.49		17.53	24.41			13.88	19.10	60.84	72.04	2.07	3.12	98.79	122.92
U_2	-25.23	-12.12		43.99	46.44			34.87	38.04	140.98	171.17	5.61	8.41	218.51	278.55
U_3	-36.67	+15.90		63.54	65.50			65.74	64.65	251.00	313.73	11.65	17.47	395.77	496.65
U_4	-44.95	+45.74		130.00	74.74			99.75	97.35	360.50	471.60	15.80	23.70	598.54	756.15
U_5	-82.83	+66.81		139.07	58.01			98.32	105.54	355.30	496.20	26.12	39.15	597.99	834.97
L_1	-345.85	-61.58	-49.20 + 0	0	118.21	0	54.50	0	61.59	0	556.45	8.35	5.56	5.20	997.24
L_2	-311.22	-51.77	-78.80 +49.20	9.08	110.34	54.50	87.20	8.14	73.08	19.61	520.80	11.91	7.95	62.06	1006.79
L_3	-284.58	-52.95	-101.80 +78.80	22.29	121.40	87.20	112.70	18.67	91.05	65.27	512.61	16.36	11.24	159.71	1051.60
L_4	-245.49	-66.80	-111.50 +101.80	35.76	150.95	112.70	123.60	40.61	118.34	152.20	543.37	26.12	17.40	335.29	1152.01
L_5	-208.86	-93.68	-114.60 +111.50	39.40	188.72	123.60	127.00	61.82	149.01	251.05	582.85	38.00	25.15	532.70	1267.18
V_0	-76.75	-44.86		64.50	96.26			24.23	69.33	106.21	234.73	3.63	5.45	212.57	463.85
V_1	-65.30	-18.83		30.10	69.39			22.93	48.97	92.16	196.40	4.06	6.09	156.28	352.87
V_2	-61.48	-1.22		28.84	36.80			22.42	47.20	81.03	184.33	4.40	6.60	112.32	302.98
V_3	-62.90	+7.66		25.37	20.86			16.05	43.47	65.93	168.05	4.19	6.28	81.83	275.31
V_4	-57.71	+4.92		15.86	14.19			10.34	35.55	50.83	142.54	2.16	3.23	51.71	226.89
V_5	-47.90	-1.50		0	5.90			0	33.00	0	79.50	0	0	0	143.10
D_1	+11.46	+8.81		49.13	35.48			38.44	27.96	144.99	122.42	6.27	4.19	252.33	198.78
D_2	+14.89	-6.20		33.94	40.65			41.54	32.36	155.00	126.87	8.06	5.18	271.77	198.07
D_3	+24.85	-20.02		23.83	49.00			49.44	31.83	176.38	137.80	11.22	7.48	304.52	220.30
D_4	+34.49	-32.40		20.36	62.33			63.04	38.50	223.01	166.95	15.05	10.00	389.33	271.74
D_5	+17.72	-15.69		23.54	50.40			64.60	40.39	235.35	203.16	8.95	5.97	371.53	295.91

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Table XXVII. Design of Members

Mem.	Stress	Composition	Area	d	Ad²	I	Mom. of In.	r	Unit Stress	Safe Stress Com.	Net Area	Safe Tens. Str.
U_1	+178820	1 cov. pl. 27"×3/8"	10.12	8.40	713	0						
	-201950	2 ∠s 4"×4"×3/8"	5.74	7.07	285	9						
		2 web pls. 20"×1/2"	20.00	1.79	64	667						
		2 ∠s 6"×6"×3/8"	8.72	10.15	898	31	2667	7.74	11100	49500		
U_2	+423320	1 cov. pl. 27"×3/8"	10.12	8.40	713	0						
	-483360	2 ∠s 4"×4"×3/8"	5.74	7.07	285	9						
		2 web pls. 20"×1/2"	20.00	1.79	64	667						
		2 ∠s 6"×6"×3/8"	8.72	10.15	898	31	2667	7.74	11100	49500		
U_3	+742390	1 cov. pl. 32"×3/8"	12.00	9.34	1047	0						
	-843270	2 ∠s 6"×6"×9/16"	12.86	7.44	712	44						
		2 web pls. 20"×3/4"	30.00	.85	22	1000						
		2 ∠s 8"×8"×11/16"	21.06	8.60	1557	129	4511	7.71	11100	84300	64.11	83500
U_4	+1077330	1 cov. pl. 33"×9/16"	18.56	9.60	1780	0						
	-1234980	2 ∠s 6"×6"×11/16"	16.16	7.65	1063	60						
		4 web pls. 20"×1/2"	40.00	.48	9	1333						
		2 ∠s 8"×8"×3/4"	22.86	8.20	1537	139						
		2 flats 9½"×9/16"	10.68	10.76	1237	0	7159	8.05	11180	1234000	93.80	1220000
U_5	+1076380	1 cov. pl. 33"×9/16"	18.56	9.83	1794	0						
	-1313360	2 ∠s 6"×6"×11/16"	16.16	7.75	1091	60						
		2 web pls. 20"×5/8"	25.00	.45	5	833						
		2 web pls. 20"×9/16"	22.50	.45	5	750						
		2 ∠s 8"×8"×3/4"	22.86	8.17	1526	139						
		2 flats 9½"×9/16"	10.68	10.73	1231	0	7434	7.94	11170	1315000	99.80	1296000
L_1	+3360	4 ∠s 8"×8"×13/16"	49.36	11.20	6188	290						
	-1001400	2 web pls. 27"×3/4"	40.50			2460	8947	9.99	11170	1004000		
L_2	+111710	4 ∠s 8"×8"×7/8"	52.92	11.18	6633	318						
	-1056440	2 web pls. 27"×3/4"	40.50			2460	9413	10.04	11310	1056000		
L_3	+287480	4 ∠s 8"×8"×13/16"	49.36	11.20	61.88	299						
	-1179370	4 web pls. 27"×1/2"	54.00			3280	9767	9.73	11370	1176000	86.80	1156000
L_4	+603520	4 ∠s 8"×8"×3/4"	45.76	11.22	57.50	277						
	-1420240	4 web pls. 27"×9/16"	60.76			3961						
		4 flats 9½"×7/16"	16.20	13.72	34.46	2	13168	10.35	11500	1413000	105.97	1380000
L_5	+958860	4 ∠s 8"×8"×13/16"	49.36	11.20	6188	289						
	-1693340	4 web pls. 27"×3/4"	81.00			4921						
		4 flats 9½"×7/16"	15.20	13.72	3446	2	14858	10.06	11540	1695000	126.31	1640000
V_0	-633910	4 ∠s 4"×4"×5/8"	18.44	9.77	1757	27						
	+382630	4 web pls. 22"×9/16"	49.50			1997	3781	7.47	9350	636000		
V_1	+282300	4 ∠s 4"×4"×9/16"	16.72	6.29	662	24						
	-477690	4 web pls. 15"×5/8"	37.50			703	1382	5.06	8770	476000		
V_2	+202180	4 ∠s 4"×4"×9/16"	16.72	6.29	662	24						
	-392840	2 web pls. 15"×15/16"	28.13			527	1213	5.20	8850	397000		
V_3	+147290	4 ∠s 4"×4"×5/8"	18.44	6.27	723	27						
	-340770	2 web pls. 15"×9/16"	16.88			316	1066	5.00	9830	347000		
V_4	-26860	2 15", 45# ⊏	26.48					5.32	10540	283000		
V_5	-143100	2 15" 33# ⊏	19.80					5.62	8316	164500		
D_1	+411350	4 ∠s 4"×4"×3/4"	21.76	6.23	845	31						
	-357800	2 web pls. 15"×5/8"	18.75			352	1228	5.51	9510	385000	31.51	
D_2	+410230	4 ∠s 4"×4"×3/4"	21.76	6.23	845	31						
	-346530	2 web pls. 15"×5/8"	18.75			352	1228	5.51	8370	364000	31.51	
D_3	+480760	4 ∠s 4"×4"×13/16"	23.36	6.25	900	33						
	-396540	2 web pls. 15"×15/16"	28.13			527	1460	5.33	9640	496000	37.43	
D_4	+606720	4 ∠s 4"×4"×13/16"	23.36	6.25	900	33						
	-489130	4 web pls. 15"×11/16"	41.25			773	1706	5.15	9880	638000	47.11	
D_5	+608260	4 ∠s 4"×4"×13/16"	23.36	6.25	900	33						
	-532640	4 web pls. 15"×11/16"	41.25			773	1706	5.15	10000	646000	47.11	

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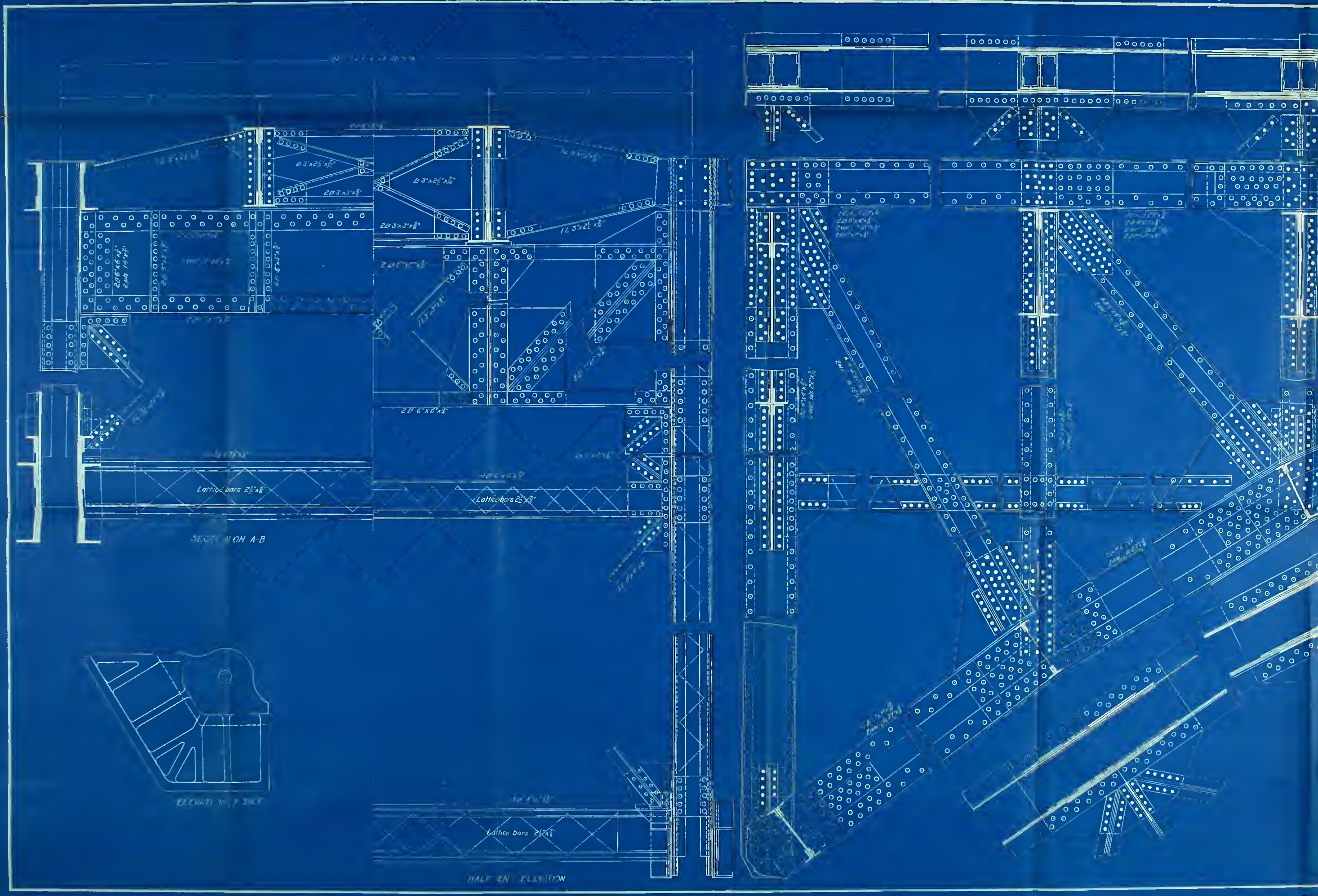
Lattice bars
Lattice bars
Lattice bars
ON A-B

CIVIL ENGINEERING DEPARTMENT

ARMOUR INSTITUTE OF TECHNOLOGY

GENERAL DRAWING

OF

207' SPAN TWO-HINGED SPANDREL-BRACED RAILROAD ARCH BRIDGE

Scale: Outline, 1/8"=1'
Details 3/8"=1'

Thesis of Roe L. Stevens
William H. Tainrus, Jr.

JUNE 3, 19[illegible]